BROCH, E. Unlined high pressure tunnels and air cushion surge chambers. *AITES-ITA 2000 World Tunnel Congress*. Durban, South African Institute of Mining and Metallurgy, 2000.

Unlined high pressure tunnels and air cushion surge chambers

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More than 80 unlined pressure shafts and tunnels with maximum water heads varying between 150 and 1000 m are in operation in Norway today. The majority of these were constructed during the years 1960–90. The oldest ones have, however, been in operation for 80 years. The development of the general layout for hydropower plants as well as the different design criteria for unlined pressure tunnels are described. Experience from the operation is discussed. Operational experience from ten unlined air cushion surge chambers have been examined with respect to air leakage. Internal air pressure varies from 19 to 78 bars and chamber volumes from 2.000 to 120.000 m^3.

Introduction

Topographical conditions in Norway are especially favourable for the development of hydroelectric energy. More than 99% of a total annual production of 120 TWh of electric energy is generated from hydropower. Figure 1 shows the installed production capacity of Norwegian hydroelectric power stations. It is interesting to note that, since 1950, underground powerhouses are predominant. In fact, of the world's 500 underground powerhouses almost one-half, i.e. 200, are located in Norway. Another proof that the Norwegian electricity industry is an 'underground industry' is that today it has more than 4000 km of tunnels. Since 1960 an average of 100 km of tunnels have been excavated every year, see Figure 1.

Throughout the design, construction, and operation of all these tunnels and underground powerhouses, valuable experience has been gained. Also, special techniques and design concepts have been developed over the years. One such Norwegian speciality are the unlined, high-pressure tunnels and shafts. Another is the so-called air cushion surge chamber which replaces the conventional vented surge chamber. It should be mentioned, as a preliminary matter, that the rock of Norway is of Precambrian and Paleozoic age. Although there is a wide variety of rock types, highly metamorphic rocks predominate. From an engineering point of view they may, in general, be classified as typical hard rocks.

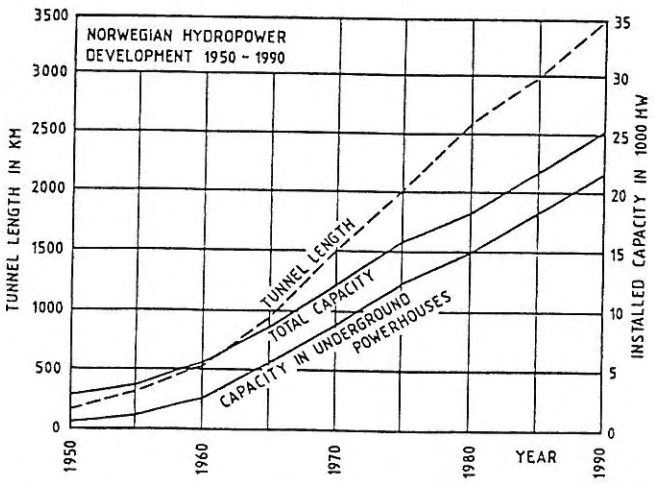

Figure 1. The development of Norwegian hydroelectric power production capacity and the accumulated length of tunnels excavated for the period 1950–1990

Most of the Norwegian hydropower tunnels have only 2–4% concrete or shotcrete lining; only in a few cases has it been necessary to increase this to 40–60%. The low percentage of lining is due not only to favourable tunnelling conditions—it is first and foremost the consequence of a support philosophy which accepts some falling rock during the operation period of a water tunnel. As long as rockfalls in certain parts of the tunnel don't occur frequently and increase the head loss, a reasonable number of small blockages spread out along the tunnel will not harm the tunnel or disturb the operation of the hydro power station. If necessary, they may be removed during later inspection and maintenance.

Early unlined pressure shafts

During and shortly after the First World War there was a shortage of steel leading to uncertain delivery and very high prices. As a result, four Norwegian hydropower stations with unlined pressure shafts were put into operation during the years 1919–21. The water heads varied from 72 to 152 m. One (Skar) was a complete failure due to too low an overburden of rock, only 22 m where the water head was 116 m. One (Toklev) has operated without any problems ever since it was first commissioned.

The Svelgen hydropower station, with a water head of 152 m, had some minor leakage during the first filling. A short section of the shaft was lined with concrete and grouted with cement. Since then the shaft has operated without problems. The fourth station, Herlandsfoss, had a 175 m long, horizontal high-pressure tunnel with water head of 136 m. Leakage occurred in an area of low

overburden, 35–40 m, and the penstock had to be extended through the whole tunnel to the foot of the inclined pressure shaft. The shaft itself had no leaks. Further details in Selmer-Olsen[1970] and Broch[1982].

Development of the general plant layout

Although three out of four pressure shafts constructed around 1920 were operating without problems after some initial problems had been solved, it took almost 40 years for the record of 152 m of water head in unlined rock at Svelgen to be beaten. Through 1958, nine more unlined pressure shafts were constructed, but all had water heads below 100 m. Before 1950 the above-ground powerhouse with penstock was the conventional layout for hydropower plants as demonstrated in Figure 2.

When the hydropower industry for safety reasons went underground in the early 1950s, they brought the steel pipes with them. Thus, for a decade or so most pressure shafts were steel-lined. During the period 1950–65, a total of 36 steel-lined shafts with heads varying from 50 to 967 m (with an average of 310 m) were constructed.

The new record shaft of 286 m at Tafjord K3, which was put into operation successfully in 1958, gave the industry new confidence in unlined shafts. As Figure 3 shows, new unlined shafts were constructed in the early 1960s and since 1965 unlined pressure shafts have been the conventional solution. Today more than 80 unlined high-pressure shafts or tunnels with water heads above 150 m are successfully operating in Norway, the highest head being almost 1000 m. Figure 3 clearly demonstrates that increasing water heads reflect an increasing confidence in unlined pressure shafts.

The confidence in the tightness of unlined rock masses increased in 1973 when the first closed, unlined surge chamber with an air cushion was successfully put into service at the Driva hydroelectric power plant. This innovation in surge chamber design is described in detail by Rathe[1975]. The bottom sketch in Figure 2 shows how the new design influences the general layout of a hydropower plant. The steeply inclined pressure shaft, normally at 45°, is replaced by a slightly inclined tunnel, 1:10–1:15. Instead of the conventional vented surge chamber near the top of the pressure shaft, a closed chamber is excavated somewhere along the high-pressure tunnel, preferably not too far from the powerhouse. After the tunnel system is filled with water, compressed air is pumped into the surge chamber. This compressed air acts as a cushion to reduce the water hammer effect on the hydraulic machinery and the waterways, and also ensures the stability of the hydraulic system.

The first design criteria

In the years before 1970 the rule of thumb for planning unlined pressure shafts in Norway was connected with the general layout for hydropower plants used at the time (Figure 2). For construction reasons the inclination of the unlined shafts varied between 31° and 47°, with 45° as the most common. The rule was expressed as follows, see Figure 4:

$h > c \cdot H$ for every point of the shaft or tunnel,

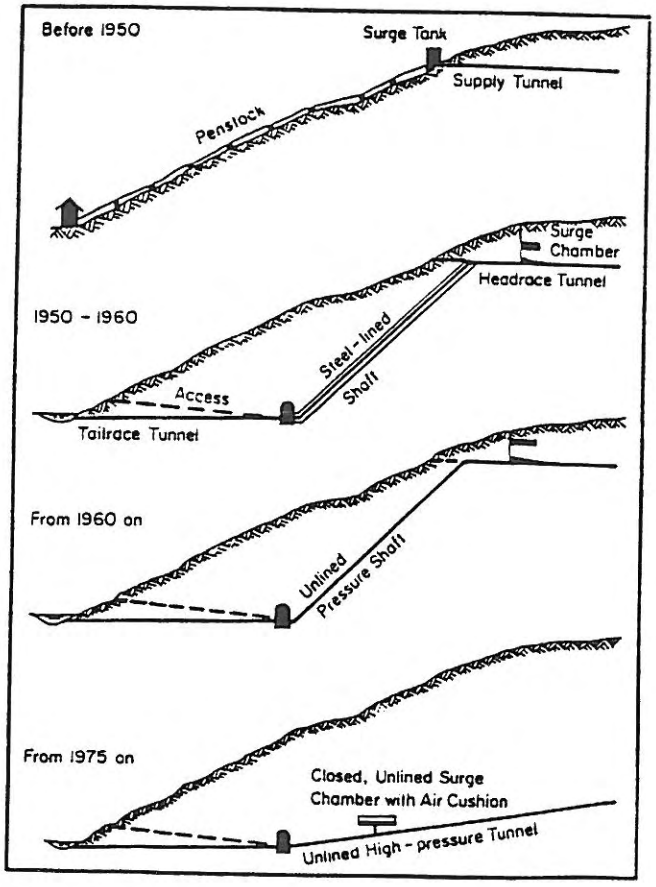

Figure 2. The development of the general layout of hydroelectric plants in Norway

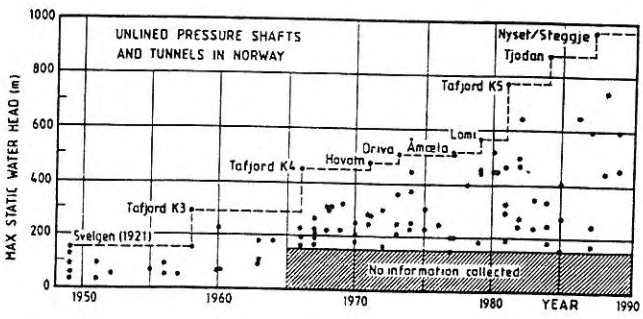

Figure 3. The development of unlined pressure shafts and tunnels in Norway

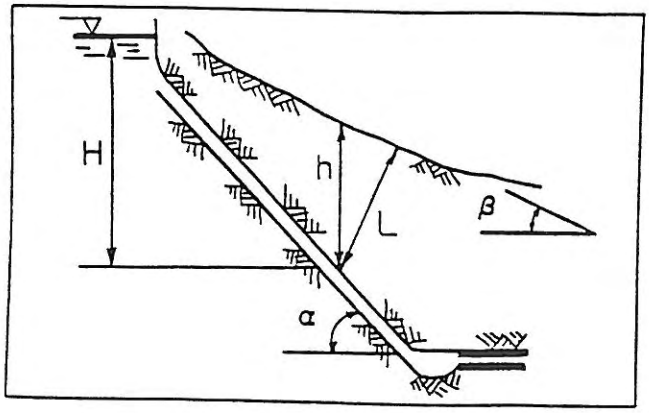

Figure 4. Definitions for the rule of thumb for high-pressure tunnels or shafts

where

h = vertical depth of the point studied (in m),
H = static water head (in m) at the point studied, and
c = a constant, which was 0.6 for valley sides with inclinations up to 35° and increased to 1.0 for valley sides of 60°.

High valley sides steeper than 60° are rather uncommon in Norway. This simple rule was, of course, to be used with care under special geological conditions.

In 1968, the unlined pressure shaft at Byrte, 300 m, failed and flooded the powerhouse. For the first time a finite element model was used to study the stresses in the rock masses surrounding the shaft, Brekke *et al.*[1970].

In the fall of 1970 another less serious failure occurred at Åskora, where an unlined tunnel in sandstone with a water head of approximately 200 m was hydraulically split. The split followed sand-filled, steeply dipping joints with a strike parallel to the very steep valley side (55°) and normal to the tunnel. The failure is described in detail by Bergh-Christensen[1975].

After this failure a new rule of thumb was introduced by Bergh-Christensen and Dannevig[1971], where the inclination of the valley side was taken directly into account (Figure 4):

$$L > \frac{\gamma_w \cdot H}{\gamma_r \cdot \cos\beta}$$

where

L = shortest distance between the surface and the point studied (in m), and
β = average inclination of the valley side.
γ_w = density of water
γ_r = density of rock.

Based on this formula, a diagram showing existing unlined pressure shafts with or without leakage was presented. This was further supplied with information by the Norwegian Geotechnical Institute[1972] and is shown in a slightly revised version in Figure 5. It is worth noticing that the unlined pressure shafts where leakage is observed are, with the exception of Bjerka, plotted below the curves defined by the rule of thumb. At Bjerka unfavourable geological conditions with steeply dipping, permeable joints with strike parallel to the valley side caused leakage at a distance up to 1 km from the pressure tunnel.

Both the old rule of thumb and the diagram in Figure 5 are still being used for quick, preliminary estimations.

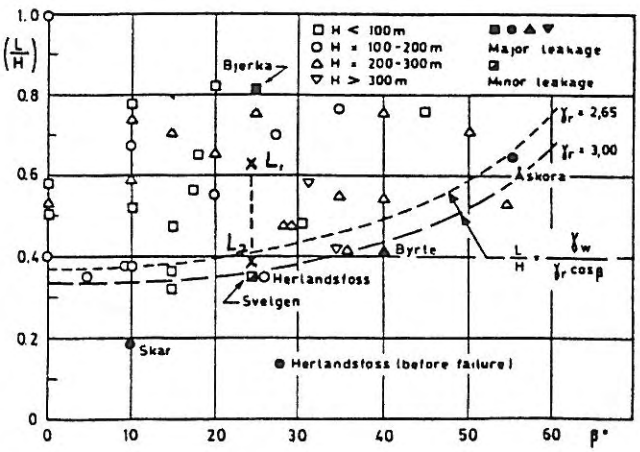

Figure 5. Unlined pressure shafts in valley sides with various inclinations, β°

Design charts based on finite element models

Parallel with the revisions of the rule of thumb, the search for better and more general design criteria was intensified at the Department of Geology, the Norwegian University of Science & Technology in Trondheim. These criteria were to be valid for unlined pressure shafts and tunnels, and also for unlined surge chambers with compressed air cushions. The first hydropower plant with this new design, Driva, was already under construction.

The new design tool was taken into use in 1971-72 and is described in detail by Selmer-Olsen[1974] and Broch[1982]. It is based on the use of computerized finite element models (FEM) and the concept is that nowhere along an unlined pressure shaft or tunnel should the internal water pressure exceed the minor principal stress in the surrounding rock mass.

Very briefly, the FEM models are based on plain strain analysis. Horizontal stresses (tectonic plus gravitational) increasing linearly with depth, are applied. Bending forces in the model are avoided by making the valley small in relation to the whole model. If required, clay gouges (crushed zones containing clay) may be introduced.

In addition to real cases, a number of idealized but typical valley sides have been analysed. One example of an idealised model is shown in Figure 6.

To make the model dimensionless, the static water pressure is expressed as the ratio H/d, where the water head is expressed as a height in the same units as the valley depth (e.g., in metres). The curved lines run through points where the internal water pressure in a shaft equals the minor principal stresses in the surrounding rock mass ($\sigma_3 = H$).

The use of the design charts can be illustrated by an example. Let the bottom of the valley, where the power station is located, be situated 100 m.a.s.l. and the top of the valley side 600 m.a.s.l. This makes d = 500 m. The maximum water level in the intake reservoir is 390 m.a.s.l. This makes H = 290 and the H/d ratio = 0.58. At all points inside or below the 0.58 line the minor principal stress in the rock mass exceeds the water pressure in an unlined shaft; hence, no hydraulic splitting should occur. If a factor of safety of 1.2 is introduced, the critical line will be the 1.2 x 0.58 = 0.7 line. As a demonstration, a 45° inclined shaft is placed in this position in Figure 6. A further discussion of how the input data may influence the results is given in Broch[1982].

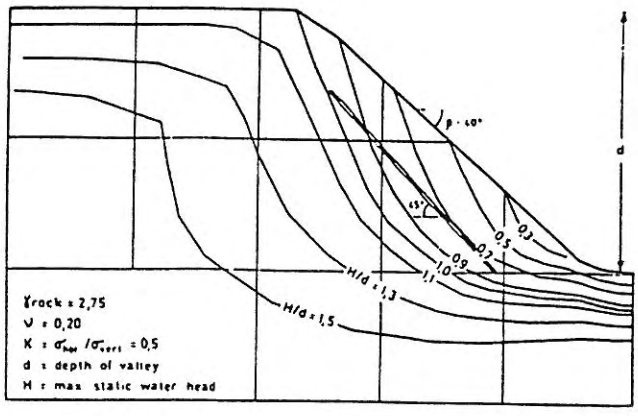

Figure 6. Design chart for unlined pressure shafts based on a finite element model. The curves run through points where the internal water pressure in the shaft equals the minor principal stress in the surrounding rock mass, H = σ_3

Evaluation of the topography

Whichever of the described methods is chosen, a careful evaluation of the topography in the vicinity of the pressure tunnel or shaft is necessary. This is particularly important in non-glaciated, mountainous areas, where streams and creeks have eroded deep and irregular gullies and ravines in the valley sides. The remaining ridges, or so-called noses, between such deep ravines will, to a large extent, be stress relieved. They should, therefore, be ignored when the necessary overburden for unlined pressure shafts or tunnels is measured. This does not mean that pressure tunnels should not be running under ridges or noses—only that the extra overburden this may give should not be accounted for in the design, unless the stress field is verified through *in situ* measurements.

The best way to explain the design approach is probably to describe an example, Broch[1984]. The simplified map in Figure 7 is based on an actual case in a tropical environment. The contour lines show parts of a very uneven valley side with a number of small creeks making deep cuts into the rock mass. As can be seen on the map, the tunnel system which consists of an upper and a lower headrace tunnel and a vertical shaft, runs, to a large extent, under a ridge. This is a good location for the unlined tunnels, but can, as will be shown, easily lead to an over-estimation of the overburden.

A simplified valley side where all protruding ridges and noses are cut off, is demonstrated by the dashed lines on the map. These straightened contour lines are drawn for every 100 m. The result of this 'revision' of the topography is shown in the vertical section through the tunnel system, Figure 8. The fully drawn line shows the actual topography. The thin, dashed line represents the rock surface and the heavy, dashed line is the revised topography.

The inclination of the revised valley side increases from 10° in the lowest part to almost 30° in the upper part. Near the upper elbow of the vertical shaft the inclination is approximately 25°.

A concrete lining, even when it is reinforced, should not be regarded as a water-tight lining of a tunnel with internal water pressures of the order of magnitude which is encountered in this case. Thus there are two critical points along the waterway which have to be controlled to see that the overburden is in accordance with the criterion. The first is at the upper elbow of the shaft and the second is at the starting point of the steel lining. The calculations are shown in Figure 8. The results are plotted as crosses marked L_1 and L_2 on to the diagram in Figure 5, by using a valley side inclination of $\beta = 25°$.

It is evident from the diagram that the revision of the topography has caused the overburden ratio to move from a fairly safe and acceptable position to a point in the diagram where leakage in unlined pressure tunnels have been experienced.

In Figure 9 the revised profile (dashed line) has been fitted to an existing design chart with a valley side inclination $\beta = 25°$. Because of a major fault zone striking parallel to the valley side, the model is stopped at el. 750. From that point the terrain is fairly flat for 700 m along the tunnel before it drops down to below el. 500, where the above-ground powerhouse is situated. The valley side rises to approximately el. 2000. The uppermost part is, however, an east-west running ridge. This peak has little influence on the stresses developed in the rock masses in the valley side. A cut-off at el. 1750 has thus been provided for this model. This gives a valley with a depth D = 1750–750 = 1000 m. With a water level in the intake reservoir of 1277 m, the static water head measured with the bottom of the valley as reference datum is H = 1277–750 = 527 m. This gives an H/d-ratio of 527/1000 = 0.53. The 0.53-line is fitted into the chart. Note that this line intersects the valley side at el. 1277. Here the water head, as well as the minor principal stresses, is zero.

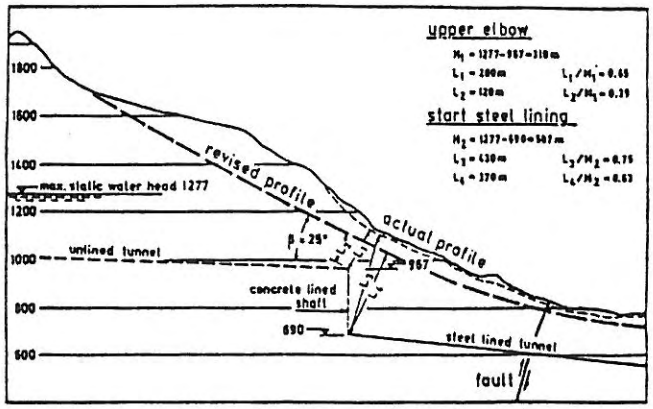

Figure 8. Vertical cross-section through the tunnel system in Figure 7. The actual topographical profile as well as the revised profile are shown. Water heads at upper elbow of the shaft, H1, and at the start of the steel lining, H2, are given. The shortest distances from these points to the profiles are shown, L1–L4 and L/H-ratios are calculated

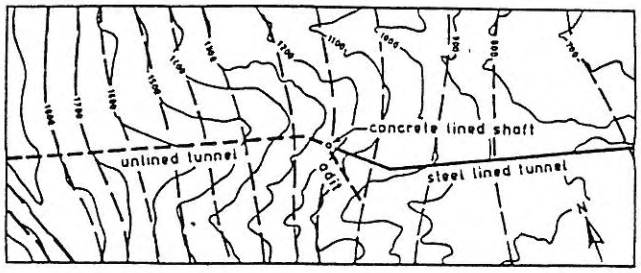

Figure 7. Tunnel system in a topographically complicated area. The height difference between contour lines is 25 m. Dashed lines are revised contour lines for every 100 m

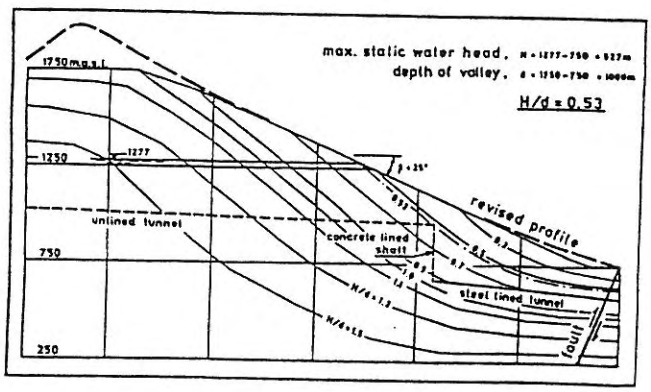

Figure 9. Design chart for unlined pressure tunnels and shafts based on stress analysis of the valley side by the use of a finite element model. Information from the example analysed in this article are fitted in using correct scale

After the revised profile of the valley side is fitted into the existing design chart model, and the actual H/d-line is drawn, the tunnel system can be plotted. This, of course, has to be done taking careful account of the scale. From Figure 9 it can be seen that the upper elbow of the shaft is slightly outside the 0.53-line. Hence, in this area there is a chance that the water may open existing joints with an orientation normal to the direction of the minor principle stress. This was exactly what happened in this particular case. Further details can be found in Broch[1984].

Geological restrictions

The FEM-developed design charts are based on the assumption that the rock mass is homogeneous and continuous, an assumption which cannot be absolutely correct even for crystalline rocks like granites and gneisses. However, observations and investigations of stress-induced stability problems such as strain bursts in a large number of tunnels in valley sides clearly indicate that the natural jointing for rock masses has only a minor influence on the distribution of the virgin stresses. Faults and weakness zones can be more important for the stress distribution as they may cause local redistribution of the stresses, Broch[1982 B].

In rock masses with alternating layers or beds of rocks with different stiffness, the stress situation may be very different from what the idealised FEM models will show. In such cases new models should be established in which the input data for the different types of rock are carefully selected. It is important to keep in mind that in a situation where there is a combination of rocks with varying stiffness, the softer rocks will take less stress than the stiffer. When a high-pressure tunnel passes through such zones of low stress rock masses, they can be over-stressed locally which may result in the opening of joints. This may lead to severe leakage.

As the permeability of rock is normally negligible, it is the jointing and the faulting of the rock mass, and in particular the type and amount of joint infilling material, that is of importance when an area is being evaluated. Calcite is easily dissolved by cold, acid water, and gouge material like silt and swelling clay are easily eroded. Crossing crushed zones or faults containing these materials should preferably be avoided. If this is not possible, a careful sealing and grouting should be carried out. Grouting is increasingly important the closer leaking joints are to the powerhouse and access tunnels and the more their directions point towards these. The same is also valid for zones or layers of porous rock or rock that is heavily jointed or broken.

For pressure shafts in valley sides, one should be warned against clay-filled, crushed zones, as they may have an unfavourable influence on the stress distribution. If they have a strike nearly parallel to the valley side with a steep or medium-steep dip towards the valley, they are especially dangerous. Not only may they change the stress distribution; they may also often cause leakage during construction as well as during operation. The hydraulic splitting of the pressure tunnels at Åskora and Bjerka was caused by such unfavourably oriented joints and faults. A careful mapping of all types of discontinuities in the rock mass is, therefore, an important part of the planning and design of unlined pressure shafts and tunnels.

During the construction period it is important that changes in all leakage into the shaft or tunnel, even minor dripping or seeping leakage, be observed. For the selection of the places for unlined air cushion surge chambers these observations are of crucial value, as these points of water leakage are openings where a loss of compressed air may take place. Core drilling and hydraulic jacking tests in the drill holes are thus normally a part of the investigation for the location of these chambers.

Hydraulic jacking tests

In later years hydraulic jacking tests have also been routinely carried out for unlined high-pressure shafts and tunnels. Such tests are particularly important in rock masses where the general knowledge of the stress situation is not well known, or difficult to interpret, based on the topographical conditions alone. The tests are normally carried out during the construction of the access tunnel to the powerhouse at the point just before the tunnel is planned to branch off to other parts of the plant as, for instance, the tailwater tunnel or the tunnel to the bottom part of the pressure shaft. See Figure 10.

To make sure that all possible joint sets are tested, holes are normally drilled in three different directions. By the use of either pre-existing or specially designed Finite Element Models the rock stress situation in the testing area as well as at the bottom of the unlined shaft are estimated. At this stage the relative values of the stresses at the two points are more important than the actual values. During the testing the water pressure in the holes is raised to a level which is 20 to 30 % higher than the water head just upstream of the steel-lining, accounting for the reduced stress level at the testing point. A complete hydraulic fracturing test is unnecessary. The crucial question is whether or not the water pressure in the unlined part of the shaft or tunnel is able to open or jack the already existing joints. Hence the importance for ensuring that all possible joint sets are tested.

If the testing shows that jacking of joints may occur, the unlined part of the waterway will have to be put further into the rock. This will normally mean that the whole powerhouse complex is moved further in. A flexible contract which allows for such changes is therefore of vital

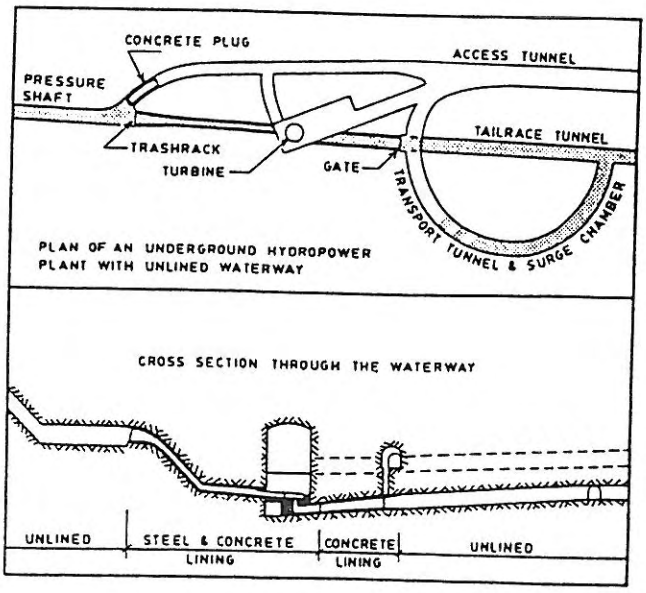

Figure 10. Plan and cross-section of an underground hydropower plant with unlined waterways

importance when unlined high-pressure shafts and tunnels are planned. Putting the powerhouse complex deeper into the rock adds length to the access tunnel, but not to the waterway.

Underground hydropower plants with unlined waterways

To demonstrate the design approach an example of an underground hydropower plant will be shown and briefly described. Figure 10 shows the simplified plan and cross-section of a small hydropower plant with only one turbine. No dimensions are given, as the intention is to show a system rather than give details. Similar layouts can be found for Norwegian plants with water heads in the range of 200–600 m.

The Figure is to some extent self-explanatory. It should be pointed out, however, that when the design charts are used, the dimensioning or critical point will normally be where the unlined pressure shaft ends and the steel lining starts. This is where the selected σ_3 = H line should intersect the waterway. The elevation of this point and the length of the steel-lined section will vary with the water head, the size and orientation of the powerhouse, and the geological conditions, in particular the character and orientation of joints and fissures. Steel lengths in the range of 30–80 m are fairly common.

The access tunnel to the foot of the unlined pressure shaft is finally plugged with concrete and a steel tube with a hatch cover. The length of this plug is normally 10–40 m, depending on the water head and geological conditions. As a rough rule of thumb the length of the concrete plug is made 4% of the water head on the plug, which theoretically gives a maximum hydraulic gradient of 25. Around the concrete plug and the upper part of the steel-lined shaft a thorough high-pressure grouting is carried out. This avoids leakage into the powerhouse and the access tunnel. Further details about the design of high-pressure concrete plugs can be found in Dahlì *et al.*[1992] or Broch[1999].

Operational experience from unlined pressure shafts and tunnels

The oldest unlined pressure shafts have now been in operation for 80 years. As mentioned previously the problems encountered during the first fillings of two of these old shafts were successfully solved, and they have been operating without unscheduled stops ever since. Three more recently constructed shafts, Byrte[1968], Bjerka[1970] and Åskora[1970] had unacceptable leakages during, or shortly after, the first filling. For all three an extension of the penstock or steel-lining solved the problem. None of the 60 pressure shafts and tunnels with water heads varying between 150 and 1000 m which have been constructed in Norway since 1970, has shown unacceptable leakage. It is thus fair to conclude that the design and construction of unlined high-pressure tunnels and shafts is a well-proven technology.

The first filling of a pressure shaft or tunnel should be done in a controlled way. The reason is that during and after the excavation of the tunnel the surrounding rock mass has gradually been drained and may even have been emptied of water. This is shown by the great reduction in the groundwater leakage into tunnels which are commonly observed over time. When the tunnel is later filled with water, the emptied joints and pores are also filled. By carrying out leakage measurements during filling, unforeseen leakage can be detected.

It is normal procedure to fill a shaft in steps or intervals of 10–30 hours, as shown for instance in Figure 11. During the intervals the water level in the shaft is continuously and accurately monitored by an extra-sensitive manometer. By deducting for the inflow of natural groundwater and the measured leakage through the concrete plug, it is possible to calculate the net leakage out from the unlined pressure tunnel or shaft to the surrounding rock masses. Some typical leakage curves are shown in Figure 12. The leakage from a tunnel is large during the first hours, but decreases rapidly and tends to reach a steady state after 12 to 24 hours, depending on the joint volume that has to be filled.

From some of the pressure tunnels and shafts where leakage measurements have been carried out, an average permeability coefficient of 1 - 10 x 10^{-9} m/s has been calculated. With this very low permeability a leakage of 0.5 –5 l/s per km has been measured, Palmstrøm[1987].

Operational experience from the air cushion surge chambers

As part of a research programme on the air and gas

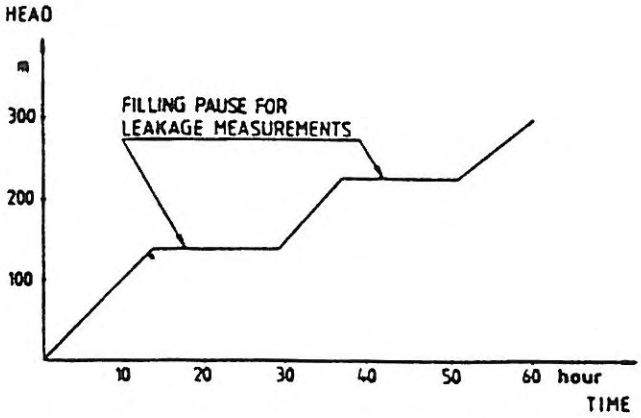

Figure 11. Example of controlled, slow filling of a pressure conduit, from Buen and Palmstrøm[1982]

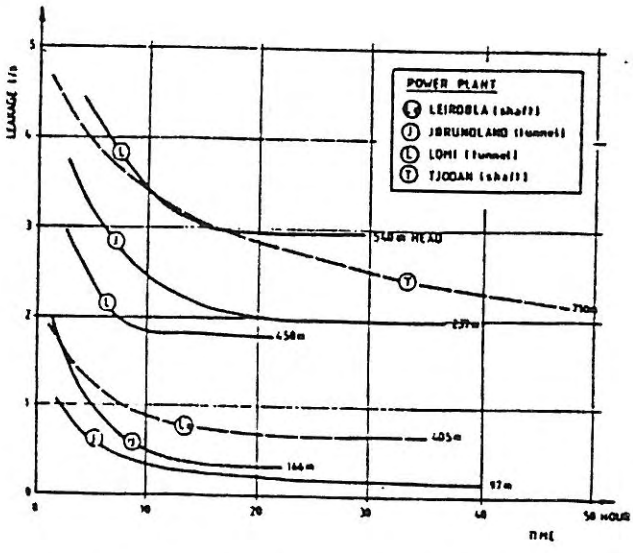

Figure 12. Measured net water leakage from various unlined high pressure shafts and tunnels, from Palmstrøm[1987]

tightness of rock masses, all ten air cushions in Norway have been carefully studied. The results are presented in a doctoral thesis by Kjørholt[1991] and in several published papers as, for instance, Goodall *et al.*[1988], Kjørholt and Broch[1992] and Kjørholt *et al.*[1992]. The last-mentioned paper is especially devoted to the geotechnical design of air cushions.

Air cushion surge chambers are used as an economical alternative to the traditional open surge shaft for damping of headrace tunnel transients from changes in power plant loading. As illustrated in Figure 13, the air cushion surge chamber is a rock cavern excavated adjacent to the headrace tunnel, in which an air pocket is trapped. The surge chamber is hydraulically connected to the headrace tunnel by a short (<100 m) tunnel. This solution gives substantial freedom in the lay-out of the tunnel-system and the siting of the plant. Schemes which have used air cushions have tended to slope the headrace tunnel directly from the reservoir towards the power station as indicated in Figure 2. Air cushions are also favoured where the hydraulic head of the headrace is above ground surface. In such cases, construction of an open surge shaft would require erection of a surge tower.

To avoid air-escape from the surge chambers, they are located a few metres above the roof of the nearby headrace tunnel as shown in Figure 13. Normally the total cavern volume needs to be 50% larger than the necessary air volume. The shape of the surge chamber is not a crucial point. Both long caverns as well as ring shaped caverns are in use.

Some basic data about the air cushion surge chambers are given in Tables I and II. As can be seen different gneisses are the dominating type of rock. Jointing is in general, low to moderate and the estimated permeability is low, not to say very low. It should, however, be kept in mind that the positions for these air cushion surge chambers are carefully selected to give as favourable rock mass conditions as possible. The final choice is often based on supplementary hydro-geological data obtained by exploratory drilling. Hydraulic jacking tests or rock stress measurements are used when deemed appropriate to verify that the stress conditions are favourable to the concept.

The surge chambers are typically connected by 50 mm diameter pipelines to one or more compressors which are located in nearby access tunnels at atmospheric pressure.

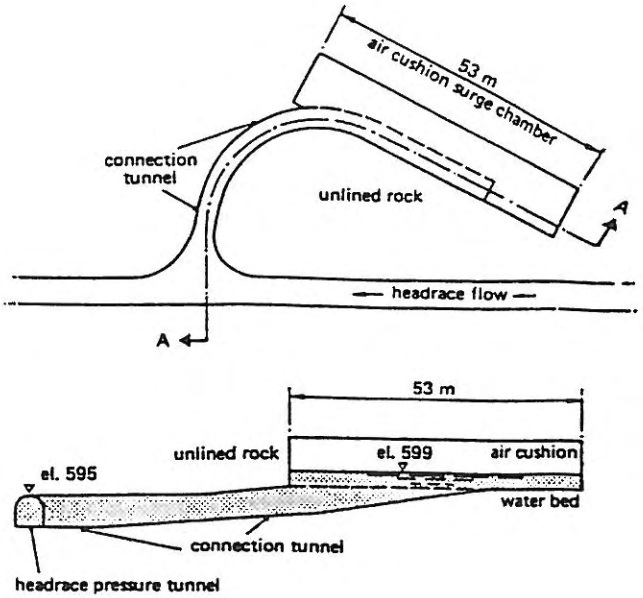

Figure 13. Plan and profile of the Ulset air cushion surge chamber

Total compressor capacities, range from 180 Nm³/h at Jukla to 2.300 Nm³ at Osa, the latter being atypically high.

Only the Tafjord site has experienced an unacceptably high leakage rate. Air losses have been reduced by remedial works at Kvilldal and Osa to allow safer and freer operation of the facilities. The history of those sites is discussed in more detail below.

At the other seven sites, the compressors are typically used on less than two per cent duty cycles for maintaining the cushion air volume. Filling time varies with compressor capacity, air cushion volume and leakage, but normally ranges from five to fifty days. Emptying of the headrace tunnel and surge chamber is normally conducted at time intervals longer than one year for maintenance such as cleaning of trash racks, inspections and repair.

Shutdown of the system requires from two to ten days. Air is released from the surge chamber prior to closing of the waterway gates. Incomplete emptying of the surge chamber has resulted in air pocket expansion and formation

Table I
Features of air cushion sites

Site	Rock type	Natural rock permeability* (m^2)	Ratio between air cushion pressure and natural groundwater pressure	Air leakage (Nm^3/h)	Air leakage (% / day)
Driva	banded gneiss	no data	0.6–0.7	0	0
Jukla	granitic gneiss	$1 \cdot 10^{-17}$	0.2–0.7	0	0
Oksla	granitic gneiss	$3 \cdot 10^{-18}$	1.0–1.2	< 5	< 0.01
Sima	granitic gneiss	$3 \cdot 10^{-18}$	0.8–1.2	< 2	< 0.01
Osa	gneissic granite	$5 \cdot 10^{-15}$	13	900/70**	11/1.0
Kvilldal	migmatitic gneiss	$2 \cdot 10^{-16}$	> 1.0	240/0***	0.2/0
Tafjord	banded gneiss	$3 \cdot 10^{-16}$	1.8–2.1	150/0***	5/0
Brattset	phyllite	$2 \cdot 10^{-17}$	1.5–1.6	11	0.2
Ulset	mica gneiss	no data	1.0–1.2	0	0
Torpa	meta siltstone	$5 \cdot 10^{-16}$	1.7–2.0	400/0***	2.0/0

*to obtain hydraulic conductivity in m/s, multiply by $\propto 10^7$
**before/after grouting
***without/with water curtain in operation

Table II
Air cushion features

Site	Commissioning date	Powerplant capacity (MW)	Distance to turbine (m)	Conect. tunnel length (m)	Vertical cavern cross-sect. (m^2)	Installed compressor capacity (Nm^3/h)	Ratio between max. air cushion head and min. rock cover (m/m)
Driva	1973	140	1300	20	111	425	0.5
Jukla	1974	35	680	40	129	180	0.7
Oksla	1980	206	350	60	235	290	1.0
Sima	1980	500	1300	70	173	270	1.1
Osa	1981	90	1050	80	176	2320	1.3
Kvilldal	1981	1240	600	70	260–370	500	0.8
Tafjord	1982	82	150	50	130	260	1.8
Brattset	1982	80	400	25	89	700	1.6
Ulset	1985	37	360	40	92	360	1.1
Torpa	1989	150	350	70	95	940	2.0

of ice in at least two of the surge chambers (Sima and Brattset). The existence of such freezing conditions at other sites must also be assumed.

Of all the surge chambers, **OSA** is situated at the shallowest depth (145 m). The surge chamber was the preferred alternative to a 40 m high surge tower at surface. The Osa site is the first case where excessive air loss was observed. Shortly after commissioning, an air leakage of about 900 Nm^3/h developed. This leakage represented about 38 per cent of the installed compressor capacity and was considered unacceptable for operational safety and high energy consumption.

After less than one year of operation the decision was made to repair the air cushion surge chamber. Grouting was undertaken at pressures as high as 4.0 Mpa. A total of 36 tons of cement and 5500 l of chemical grout were injected. This grouting programme focused on areas where leakage was indicated by water inflow. As a result of the grouting, air loss from the cavern was dramatically reduced from 900 Nm^3/h to 100 Nm^3/h initially and to 70 Nm^3/h in the long term, which is managed comfortably by the compressor plant.

At **KVILLDAL**, all leakage through the rock appears to have been eliminated by installing a water curtain. The initial air loss was close to 60 per cent of stored volume per year. Kvilldal was the first site where artificial hydrodynamic control, i.e. a water curtain, was used.

The surge chamber at Kvilldal is by far the largest with a working air volume of about 80.000 m^3. To achieve such a large volume a rectangular ring tunnel shape was selected, see Figure 14. The overall configuration of the surge chamber was selected on the basis of construction feasibility and was also found to be hydraulically suitable. The chamber location was specifically chosen to avoid fractured zones in the rock on the basis of 1.600 linear metres of drillhole. As a result of this careful pre-investigation, the quality of the host rock is excellent at the selected location, and no rock bolts, grouting or other support measures were needed.

Air loss from the Kvilldal surge chamber in the first year of operation was 250 Nm^3/h, but without evidence of concentrated leakage at surface. Because such a loss tested compressor capacity, a water curtain was subsequently installed. The water curtain consisted of a fan of about forty-five 50 mm diameter bore holes with a total length of about 2.500 m and a distance between the water curtain and the chamber varying from about 10 to 20 m.

The water curtain is supplied by a 120 l/min. capacity water pump. The maximum water pressure used in the water curtain is 5.1 MPa which is 1 MPa above the pressure in the chamber. At this pressure the average water flow to the curtain is 32 l/min. Air loss since installation of the water curtain has decreased by more than an order of magnitude (Table II).

The **TAFJORD** air cushion was first commissioned without any leakage-preventing measures being taken. But, even though the leakage at this site was somewhat less than at Kvilldal, the compressors did not have sufficient capacity. The surge chamber at Tafjord was therefore out of operation from 1982 to1990 (i.e. the cavern was completely water filled). An attempt to grout a major fracture intersecting the cavern did not improve the leakage condition. Fortunately, the Tafjord air cushion is not crucial to the operation of the Pelton system to which it is connected. The power-plant has consequently been able to operate without a surge facility.

A water curtain was installed at Tafjord in 1990, partly as a research project. The curtain consists of 16 drill holes (diameter 56 mm) which covers both the roof and the upper part of the cavern walls. Also, at this site the air leakage disappeared when the water curtain was put in operation at

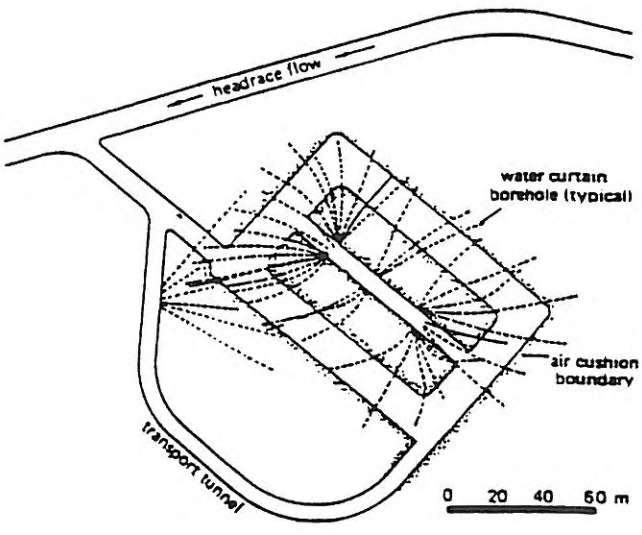

Figure 14. Plan of Kvilldal air cushion surge chamber with water curtain

the designed pressure (0.3 MPa above the air cushion pressure).

The **TORPA** air cushion is the only one where a water curtain was included in the original design. The water curtain consists of 36 bore holes (64 mm diameter), drilled from an excavated gallery 10 m above the cavern roof, (see Figure 15). In addition to the water curtain, grouting was undertaken during construction to improve the rock condition.

As for the two other water curtains, no air leakage has been registered from the air cushion when the water curtain is in operation at design pressure (0.3 MPa above the air cushion pressure). To get an idea of the air leakage potential at Torpa, the water curtain was turned off for two days. This resulted in an 'immediate' leakage rate of 400 Nm^3/h. The leakage ceased as soon as the water curtain was put in operation again. The measured leakage rate corresponds very well with results from theoretical calculations.

Air leakage through the rock masses are tabulated in Table II. Also indicated for each site is the ratio of the air cushion pressure and the natural groundwater pressure.

Six of the air cushions have a natural air leakage rate that is acceptable. Three air cushions have no air leakage at all through the rock mass. At four of the air cushion sites (Osa, Kvilldal, Tafjord and Torpa), the natural leakage rate was too high for a comfortable or economic operation, and remedial work was carried out. These four sites are located in the most permeable rock masses of all ten cushions, and also have ratios between air cushion pressure and natural groundwater pressure above 1.0.

Concluding remarks

Experience from a considerable number of pressure tunnels and shafts has been gathered over a long period in Norway. These show that, providing certain design rules are followed and certain geological and topographical conditions are avoided, unlined rock masses are able to contain water pressures up to at least 100 bars, equalling 1000 m water head.

Air cushions have proved to be an economic alternative to the traditional open surge shaft for a number of hydropower plants. Experience shows that the hydraulic design should follow the same principles as for an open surge shaft. The geotechnical design of the air cushion cavern should follow the same basic rules as for other rock caverns.

Air leakage through the rock masses is the major challenge when designing and constructing an air cushion. A certain leakage may, for economic reasons, be accepted. If, however, the leakage exceeds a given limit, both grouting and the use of water curtains are possible remedial actions. Experience has shown that grouting will reduce the leakage to a certain extent, while a water curtain is able to eliminate the leakage through the rock.

In the future this experience may also be important outside the hydropower industry, for instance in the construction of cheap, unlined storage facilities for different types of gas or liquid under pressure.

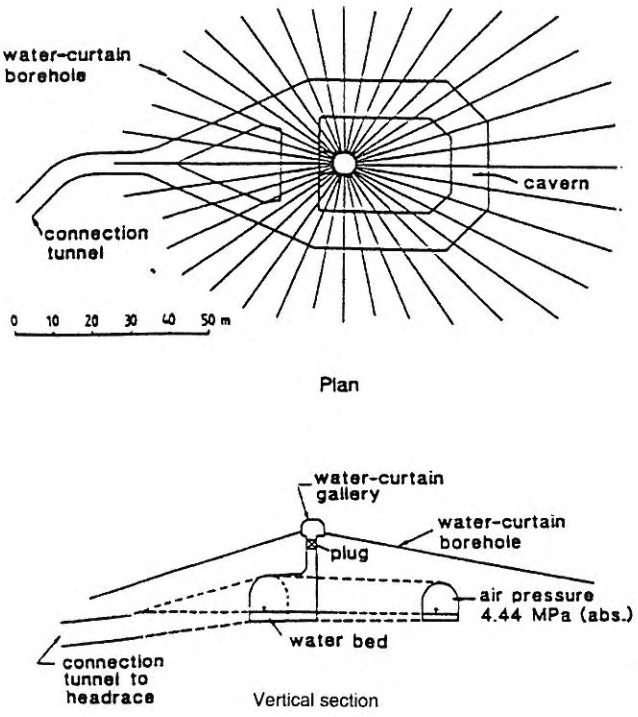

Figure 15. Geometry of Torpa air cushion surge chamber with water curtain

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